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Memorandum

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To: BARTT GUNTER, CHIEF
Structure Design
Office of Bridge Design-South 2
Bridge Design Branch 19

Attn: Mamunur Rahman

Date: September 12, 2013

File: 08-SBd-138-PM 24.2
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Horsethief Ck Br. - Replace
Ex Br. #54-0816
Proposed Br. #54-1271

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2 MS #5
Design Branch A

Subject: Foundation Report for Horsethief Creek Bridge Replacement

Pursuant to a request by the Office of Bridge Design South 2 (OBDS2), Design Branch 19, this report presents Foundation Recommendations for the proposed Horsethief Creek Bridge Replacement (Br. No. 54-1271), and supersedes all previously generated Preliminary Foundation Reports for this structure. The following foundation recommendations are based on subsurface information gathered during foundation investigations conducted in October 2012, the1969 as-built data, and plans and loads provided by OBDS2.

With regards to the current foundation recommendations, all elevations referenced in this report are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

It is proposed to replace the existing Horsethief Creek Bridge (Br.# 54-0816), built in 1968, with a new structure. The new proposed bridge will be designated as bridge number 54-1271. The project is located in San Bernardino County on State Route (SR) 138 at Post Mile 24.2.

The existing 3 span, 2 lane bridge will be replaced with a new 3 span, 2 lane bridge, to meet current seismic design standards. The existing bridge is supported on driven concrete piles. The General Plan provided by OBDS2 dated 9/4/13 proposes a deep foundation system using 48-inch and 60-inch CIDH piles, with permanent steel casings at abutments and bents respectively. The CIDH Pile diameters are reduced to 36-inch and 48-inch from the permanent casing tip elevation to the specified pile tip elevation, for abutments and bents respectively.

Geology, Subsurface Investigation and Lab Testing

The project site is located in a shallow wash that has been cut into the older alluvial valley fill and the sandstone and granitic bedrock. Large cobbles and small boulders, possibly from the nearby granitic (quartz diorite) bedrock, are represented within the recent wash alluvial matrix.

The site is located on the northern flank of the San Bernardino Mountains of the eastern Transverse Ranges Geomorphic Province near the margin with the Mojave Desert Province. The bridge project area is within the east flowing Horsethief Creek drainage area.

A subsurface investigation was conducted at the bridge site in October 2012, which consisted of 4 mud rotary borings (R-12-001 through R-12-004), with surface elevations ranging from approximate elevations of 3,274 to 3,266 feet The borings extended to a maximum depth of about 120 feet below grade; and the deepest elevation of about 3,150 ft. The Log of Test Borings (LOTB), for the recent investigation was submitted under a separate cover.

Based on the 2012 investigation, the site can be described as underlain by embankment fills for the existing bridge abutments and surficial recent alluvial sand and gravels from the Horsethief Creek wash. Layers of loose to very dense sands and silts were also encountered at depth in the test borings. Cobbles and/or boulders were also present sporadically on the surface and within the wash alluvium. These surficial deposits are underlain by a soft to moderately hard weakly cemented sandstone bedrock of the Crowder Formation; and the moderately weathered, moderately hard quartz diorite (granitic) crystalline rock of the local area. For further details please refer to the log of test boring (LOTB) sheets.

Some soil samples were evaluated in the lab for particle size analysis (PA) and corrosiveness. Unconfined Compression (UC) test were also performed on a few of the testable rock samples. The results from the PA and moisture content were incorporated into the Log of Test Boring (LOTB) sheets, and corrosion results are summarized in a latter section of this report under 'Corrosion'. The results of the UC test are summarized in Table 1 (below):

Table 1 – Unconfined Compression Test Summary

Boring	Sample Depth / Elev. (ft)	Unconfined Compressive Strength (tsf)
R-12-001	110-111.5 / 3,164 – 3,162.5	985
R-12-003	117-118 / 3,152.5 – 3,151.5	556

Please refer to the LOTBs and lab results for more details.

Also, according to a 1969 dated As-Built LOTB, two rotary borings were used to obtain subsurface information during January of 1966. The rotary borings extended to a maximum depth of about 41 feet below grade (elevation 3,194 ft). A foundation report dated 2/28/1966, describes the subsurface material at the site as follows: "The site is underlain by about 30 feet of slightly compact gravelly sand. This material is underlain by very dense sandstone to elevation 3,194, the maximum boring depth attained". For further soil and rock information about the 1966 investigation, please refer to the As-Built LOTB.

Ground Water

During the subsurface geotechnical investigation in October, 2012, the highest groundwater was measured at elevation 3,241.3 ft, in boring R-12-004. However, the groundwater elevation may fluctuate due to seasonal variation, and surface flowing in the creek.

According to the 1969 As-Built LOTB, both borings encountered ground-water at a very shallow depth of about one foot (elevation 3,233 feet) during the January 1966 field investigation.

Scour Potential

Caltrans Hydraulic Report (HR) dated January 31, 2013, indicates that the total scour depths at bents are estimated at 12 feet (Elevation of 3,231 feet). The HR (supplemented by information provided by OBDS2) indicates that the total scour depths at abutments are estimated at 2 feet (Elevations of 3,256 & 3,259 feet at Abutments 1 & 4 respectively).

Corrosion

Corrosion test results for soil samples collected from borings R-12-001 thru R-12-004 are shown below in Table 2. Since the minimum resistivity of samples is greater than 1000 ohm-cm testing for chloride and sulfates are not required. Therefore, due to the pH being greater than 5.5 for the samples tested, the site is considered to be non-corrosive based on current Caltrans' standards.

Table 2 – Corrosion Test Summary

Location	Minimum Resistivity (Ohm-Cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
Boring R-12-001 (Elev. 5-61.5ft)	12540	6.79	NA	NA
Boring R-12-002 (Elev. 40-65ft)	11300	7.63	NA	NA
Boring R-12-003 (Elev. 30-71ft)	10305	8.06	NA	NA
Boring R-12-004 (Elev. 5-65.5ft)	14885	7.71	NA	NA

Note: Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 2000 ppm or greater, or has a pH of 5.5 or less. With the exception of MSE walls, soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,000 ohm-cm.

Seismicity

Ground motion recommendations are based on the Caltrans 2009 Seismic Design Procedure (SDP) as described in the Seismic Design Criteria Version 1.6 (SDC) Appendix B, the Acceleration Response Spectrum (ARS) Online Tool v2.2.06, and 2012 Log of Test Borings (LOTB) drilled in October 2012.

Based on the available 2012 subsurface geotechnical data for the proposed replacement, the average shear wave velocity for the upper 100 feet of subsurface materials is estimated as V_{S30} = 300 m/s using the shear wave velocity correlation with Standard Penetration Test (SPT).

Design Response Spectrum

Based on the 2009 SDP, the design response spectrum is the upper envelope of the deterministic and probabilistic response, but is not less than a minimum deterministic response spectrum resulting from a Mmax = 6.5 earthquake on a vertical strike-slip fault at a distance of 7.5 miles (12 km).

The deterministic response spectrum is obtained by taking the arithmetic average of the median response spectrum calculated using the 2008 Campbell-Bozorgnia and 2008 Chiou-Youngs ground motion prediction equations. The probabilistic response spectrum is obtained for 5 percent probability of exceedance in 50 years (corresponding to approximately a 975 year return period) using the 2008 USGS Seismic Hazard Map. Adjustments to account for site conditions and fault effects were implemented.

For this site the probabilistic response spectrum controls. Caltrans' Acceleration Response Spectrum (ARS) Online Tool v2.2.06 was utilized. The calculated spectrum was also adjusted for near field effect. The corresponding peak horizontal ground acceleration at proposed site is 0.87 g. The recommended acceleration response spectrum is attached.

Liquefaction Potential Evaluation

Soil liquefaction is a phenomenon in which saturated loose to medium dense, predominantly granular soils lose most, if not all, of shear strength and stiffness due to the development of excess pore pressure when subjected to ground shaking. Effects of liquefaction on ground surface include foundation settlement and reduction in bearing capacity, sand boils, and ground settlement and lateral spreading.

The soil profile at each boring location was analyzed for liquefaction potential in accordance with the procedure suggested by Seed et al (1985) and modified by Youd et al (2001). The results of our analysis indicate that a sandy soil layer at loose to medium dense states varying from elevation 3,240 feet to 3,210 elevation are prone to liquefaction.

Seismic settlement at the site due to strong ground motion is estimated based on the procedure suggested by Tokimatsu and Seed (1987) is about 5-6 inches.

Lateral Spread

It is anticipated that the lateral spread may occur at the project site due to earthquakes. The additional lateral force generated by the lateral spread should be considered in design.

Kinematic approach is used to estimate the lateral spreading forces in this report. Kinematic approach is a displacement-controlled method. It accounts directly for the interaction between the moving soil and the pile displacement and includes the available mobilized soil resistance since it accounts for the residual strength of liquefied soils. Computer programs LPILE v5 and GSTABLE were used during analyses

The results of analyses show that to consider the effects of soil movements during earthquake, the lateral spreading force of 400 kips should be applied at cut-off elevation of each CIDH pile in abutments of this project.

Surface Fault Rupture Hazard

The site is not located within a Fault Hazard Zone and is about 1.2-mile from the nearest Caltrans-active fault (Cleghorn fault zone (Northern Cleghorn section). Therefore the potential for surface rupture / displacement hazard due to fault movements is considered low.

As-Built Foundation Data

The Foundation data shown in Table-3 are based on the 1969 dated As-Built General Plan:

Table 3 - As-Built Data for existing Bridge (54-0816)

Location	Foundation Type	Design Loading	Average Pile-Tip Elevation
Abutment 1	Driven Concrete Piles	45 ton	3,205.0
Bent 2	Driven Concrete Piles	45 ton	3,205.0
Bent 3	Driven Concrete Piles	45 ton	3,205.0
Abutment 4	Driven Concrete Piles	45 ton	3,207.0

Note: As-Built elevations refer to NGVD29 datum.

Foundation Recommendations

The following recommendations are for the proposed replacement of Horsethief Creek Bridge (Br. #54-1271) as shown on the General Plan dated September 4, 2013. A deep foundation system using 48-inch and 60-inch CIDH piles with permanent steel casings are recommended at abutments and bents respectively. The CIDH Pile diameters are reduced to 36-inch and 48-inch from the permanent casing tip elevation to the specified pile tip elevation, for abutments and bents respectively. Steel casings are recommended to facilitate construction by helping mitigate potential caving concerns.

The geotechnical pile capacity will equal or exceed the required design loads presented in following tables. The specified pile tip elevations are listed in Tables 4 and 5 for the abutments and bents respectively, and summarized in Table 6.

The general foundation information and design loads were provided by structure designers. It is shown in Tables 7 and 8. If any information or design loads differ from those described in these tables, our office should be notified and modification of our recommendations may be necessary.

Table 4 - Foundation Recommendations for Abutments

Horsethief Creek Bridge (Replace) Proposed Bridge # 54-1271									
Support	Pile	Cut-off Elevation (ft)	LRFD Service-I Limit State Load (kips) per Support		LRFD Service-I Limit State Total Load per Pile (Compression) (kips)	Required Nominal Resistance Per Pile (kips)	Design Tip Elevations (ft)	Specified Tip Elevation (ft)	Steel Casing Specified Tip Elevation (ft)
			Total	Permanent					
Abut. 1 Piles 1 & 2	48-inch CIDH Pile with Permanent Casing	3,251.25	1,271	992	287	574	3,180 (a)	3,180	3,195
Abut. 1 Piles 3, 4 & 5		3,253.25							
Abut. 4 Piles 1 & 2	48-inch CIDH Pile with Permanent Casing	3,257.25	1,403	1,118	306	612	3,185 (a)	3,185	3,205
Abut. 4 Piles 3, 4 & 5		3,259.25							

Notes: (1) Design tip elevations are controlled by: (a) Compression, (b) Tension (c) Settlement, (d) Lateral Load, respectively.
(2) CIDH Pile diameter will be reduced to 36-inch from casing tip elevation to specified tip elevation.

Table 5 - Foundation Recommendations for Bents

Horsethief Creek Bridge (Replace) Proposed Bridge # 54-1271											
Support Location	Pile Type	Cut-off – Elevation (ft)	Service-I Limit State Load per Support (kips)	Total Permissible Support Settlement (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elevations (ft)	Specified Tip Elevation (ft)	Steel Casing Specified Tip Elevation (ft)
					Strength Limit		Extreme Event				
					Comp. (φ=0.7)	Tension (φ=0.7)	Comp. (φ=1)	Lateral (φ=1)			
Bent 2	60-inch CIDH Pile with Permanent Casing	3,245	3,045	1	1,679	0	1,620	1,100	3,155 (*a-I*) 3,170 (*a-II*) 3,175 (*b-II*)	3,155	3,205
Bent 3	60-inch CIDH Pile with Permanent Casing	3,250	3,183	1	1,751	0	1,520	1,000	3,155 (*a-I*) 3,165 (*a-II*) 3,170 (*b-II*)	3,155	3,200

Notes:

1) *Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (b-I) Tension (Strength Limit), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement, (d) Lateral Load*
2) *The specified tip elevation shall not be raised above the design tip elevations for tension, lateral, and tolerable settlement.*
3) *These are only estimates based on available subsurface information. It is recommended to install casings for a minimum of 5 ft, into the top of the bedrock (Sandstone).*
4) *CIDH Pile diameter will be reduced to 48-inch from casing tip elevation to specified tip elevation.*

Table 6 - Pile Data Table

Location	Pile Type	Nominal Resistance (kips)		Cut Off Elevation (ft)	Estimated[1] Casing Tip Elevation (ft)	Design Tip Elevation (ft)	Specified Tip Elevation (ft)
		Compression	Tension				
Abut. 1 Piles 1 & 2	48-inch CIDH Pile with Permanent Casing	574	0	3,251.25	3,195	3,180	3,180
Abut. 1 Piles 3, 4 & 5				3,253.25			
Bent 2	60-inch CIDH Pile with Permanent Casing	2,400	1,100	3,245	3,205	3,155 (a-I) 3,170 (a-II) 3,175 (b-II)	3,155
Bent 3	60-inch CIDH Pile with Permanent Casing	2,500	1,000	3,250	3,200	3,155 (a-I) 3,165 (a-II) 3,170 (b-II)	3,155
Abut. 4 Piles 1 & 2	48-inch CIDH Pile with Permanent Casing	612	0	3,257.25	3,205	3,185	3,185
Abut. 4 Piles 3, 4 & 5				3,259.25			

Notes:

1) *Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (b-I) Tension (Strength Limit), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement, (d) Lateral Load*
2) *The specified tip elevation shall not be raised above the design tip elevations for tension, lateral, and tolerable settlement.*
3) *CIDH Pile diameter will be reduced to 36-inch (at Abutment locations), to 48-inch (at Bent locations) from casing tip elevation to specified tip elevation.*
4) *These are only estimates based on available subsurface information. It is recommended to install casings for a minimum of 5 ft, into the top of the bedrock (Sandstone).*

Table 7: General Foundation Information Provided by OBDS2

<table>
<tr><th colspan="10">Horsethief Creek Bridge (Replace) Proposed Bridge # 54-1271
Foundation Design Data Sheet</th></tr>
<tr><th rowspan="2">Support No.</th><th rowspan="2">Design Method</th><th rowspan="2">Pile Type CIDH (inch)</th><th rowspan="2">Finished Grade Elevation (ft)</th><th rowspan="2">Cut-off Elevation (ft)</th><th colspan="2">Pile Cap Size (ft)</th><th rowspan="2">Permissible Settlement under Service Load (inch)*</th><th rowspan="2">Number of Piles per Support</th></tr>
<tr><th>B</th><th>L</th></tr>
<tr><td>Abut 1 Piles 1 & 2</td><td rowspan="2">WSD</td><td rowspan="2">48</td><td rowspan="2">3,256</td><td>3,251.25</td><td rowspan="2">5.5</td><td rowspan="2">58.35</td><td rowspan="2">1</td><td rowspan="2">5</td></tr>
<tr><td>Abut 1 Piles 3, 4 & 5</td><td>3,253.25</td></tr>
<tr><td>Bent 2</td><td>LRFD</td><td>60</td><td>3,245</td><td>3,245</td><td>N/A</td><td>N/A</td><td>1</td><td>3</td></tr>
<tr><td>Bent 3</td><td>LRFD</td><td>60</td><td>3,250</td><td>3,250</td><td>N/A</td><td>N/A</td><td>1</td><td>3</td></tr>
<tr><td>Abut 4 Piles 1 & 2</td><td rowspan="2">WSD</td><td rowspan="2">48</td><td rowspan="2">3,261</td><td>3,257.25</td><td rowspan="2">5.5</td><td rowspan="2">74.01</td><td rowspan="2">1</td><td rowspan="2">5</td></tr>
<tr><td>Abut 4 Piles 3, 4 & 5</td><td>3,259.25</td></tr>
</table>

* *Based on Caltrans' current practice, the total permissible settlement is one inch for multi-span structures with continuous spans or multi-column bents, one inch for single span structures with diaphragm abutments, and two inches for single span structures with seat abutments. Different permissible settlement under service loads may be allowed if a structural analysis verifies that required level of serviceability is met.*

Table 8: Design Loads Provided by OBDS2

Horsethief Creek Bridge (Replace) Proposed Bridge # 54-1271 Foundation Design Data Sheet											
	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
Support No.	Per Support	Max. Per Pile	Per Support	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1	1,271	287	992	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Bent 2	3,045	1,085	2,536	4,384	1,679	0	0	1,620	1,620	1,100	1,100
Bent 3	3,183	1,161	2,635	4,591	1,751	0	0	1,520	1,520	1,000	1,000
Abut 4	1,403	306	1,118	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

General Notes:

1. All support locations are to be plotted on the plan view of the Log of Test Borings (LOTB), as stated in "Memos to Designers" 4-2. The plotting of the support locations should be made prior to the foundation review.

2. When applicable, the structure engineer shall show on the plans (in the pile data table), the design pile tip elevation required to meet the lateral load demands. If the design pile tip elevations required to meet the lateral load demands go beyond the specified pile tip elevations given within this report, the Office of Geotechnical Design-South 2, Branch A shall be contacted for further recommendations.

Construction Considerations

CIDH Piles

1. The CIDH piles should be constructed in accordance with current Caltrans Standard Specifications (Caltrans, 2010).

2. The actual groundwater elevation during construction may be different from those shown in the LOTBs, due to seasonal rainfall, surface runoff and other man-made conditions. However, considering the length of piles, contractors should be prepared to use a "Wet" construction method.

3. The contractor will need to use care while drilling the shafts for the CIDH piles. Insertion and removal of the drilling tools during the drilling process can cause excessive scouring and caving of the walls of the drilled shaft.

4. The contractor should anticipate varying rock drilling conditions (soft and hard rock drilling). The amount of difficulty the contractor will experience will be dependent upon the methods and means the contractor chooses to construct the CIDH piles within the varying earth materials. Refer to LOTBs for details.

5. To construct the CIDH piles using the slurry displacement methods, the slurry shall consist of mineral or synthetic slurry only. Water shall not be allowed to be used as slurry.

Steel Casings

The contractor should anticipate numerous difficulties while installing the permanent steel casings:

1. Caving conditions are anticipated in the alluvial materials overlying the bedrock. To facilitate construction, it is recommended to install permanent steel casings for a minimum depth of 5 ft into the top of the bedrock (Sandstone).

2. The recommended permanent steel casings are intended to minimize construction difficulties due to caving of the loose alluvial material overlying rock. The permanent steel casings will not eliminate the potential for caving within rock below the casing tip elevation. The methods and means used by the contractor to install the permanent steel casings and seal the contact between the casing tips and rock, will directly determine the construction difficulties the contractor will encounter with the overlying alluvium while excavating the CIDH pile in the underlying rock.

3. The contractor should not assume that installation of the permanent steel casing will allow the contractor to drill and place concrete for the CIDH piles in the dry. The intent of the permanent steel casing is only to minimize construction difficulties due to caving of the loose alluvial material overlying the bedrock.

4. During construction of the CIDH piles, the contractor should choose appropriately sized drilling tools so as to make sure not to damage or puncture the wall of the permanent steel casing while inserting and removing the drilling tools. Puncturing the wall of the steel casing may allow alluvium to enter the drilled hole. Alluvium flowing through the punctured casing and into the borehole could potentially cause massive subsidence at the ground surface.

5. During construction of the CIDH piles, the contractor should make sure to maintain a positive head between the ground water surface and the fluid level inside the permanent steel casing. If a positive head is not maintained, it could potentially lead to caving of the walls of the drilled hole, and construction difficulties.

The recommendations contained in this report are based on specific project information regarding design loads and structure locations that has been provided by Office of Bridge Design South 2, Branch 19. If any conceptual changes are made during final project design, the Office of Geotechnical Design South-2, should review those changes to determine if the foundation recommendations provided in this report are still applicable. Any questions regarding the above recommendations should be directed to attention of Farzad Qmehr (916) 227-4519 or Angel Perez-Cobo (916) 227-7167, Office of Geotechnical Design South-2.

Prepared by: Date: September 12, 2013

REGISTERED PROFESSIONAL ENGINEER
FARZAD QMEHR
No. C 42892
Exp. 3-31-14
CIVIL
STATE OF CALIFORNIA

Farzad Qmehr
Transportation Engineer
Geotechnical Design-South 2
Design Branch A

Attachment: ARS Curve

cc: R.E. Pending File
- Specs & Estimates - Ofelia P. Alcantara
- District 8 Project Manager – Bacson Quach
- HQ Project Coordination Engineer – Colyn Peterson
- District 8 Environmental Planning – Patraic Kelly
- District 8 Materials Engineer – Bruce Kean
- HQ GS Corporate – Shira Rajendra
- HQ Geotechnical Design South-2 – Abbas Abghari – OGDS-2
- HQ Geotechnical Design South-2 – Angel Perez-Cobo

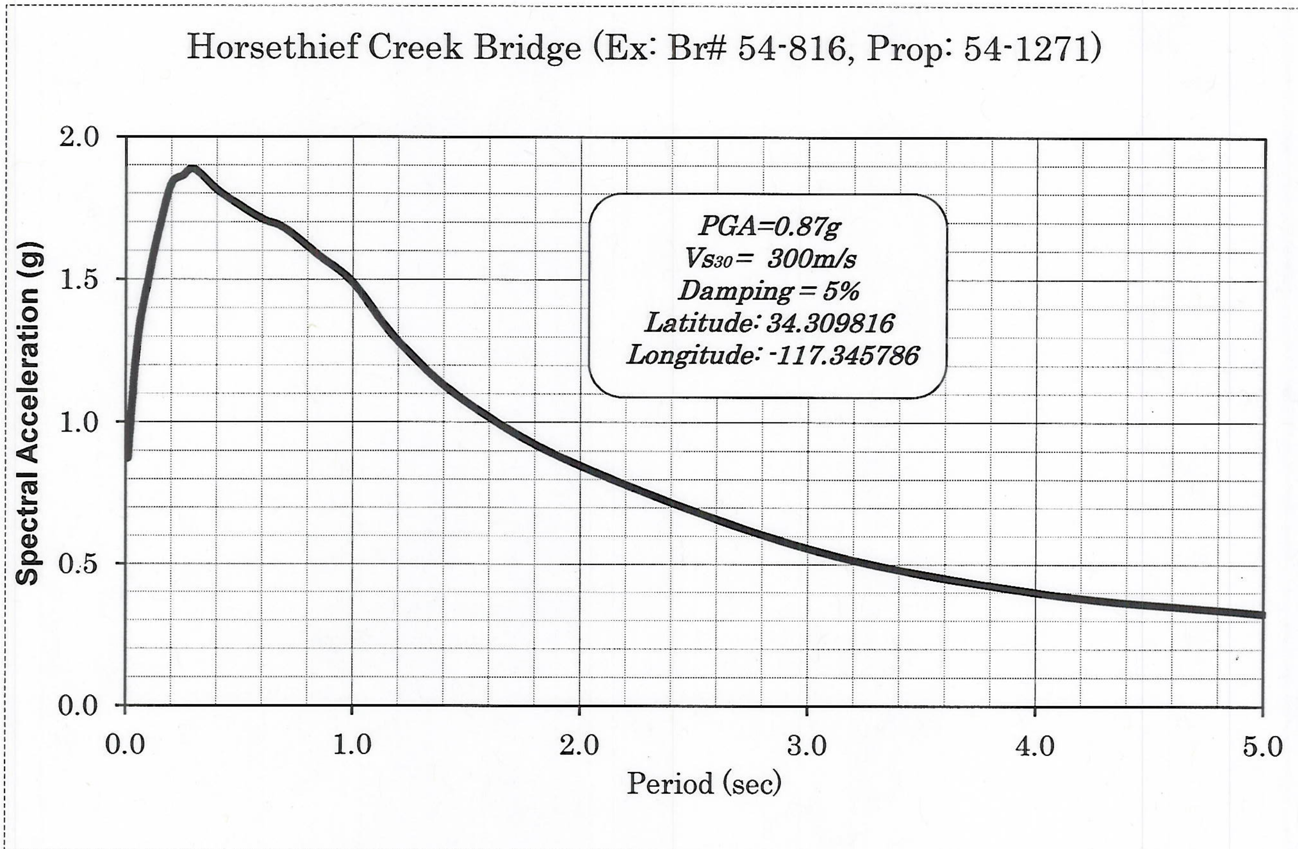

Figure1. Recommended Acceleration Response Spectrum (ARS) Curve